Relative Merits. If the shear transfer deficiency is governed by the existing nailing, the most cost-effective alternative most likely is to provide additional nailing (Technique 1); however, stripping of the flooring or roofing surface is required. If it is not feasible to provide adequate additional nailing within the length of the shear wall, the installation of a collector (Technique 2) probably will be the most cost-effective alternative. As indicated in the detail on the left of Figure 3.7.1.2b, if the nailing of the diaphragm to the new blocking is inadequate to transfer the desired shear force over the length of the shear wall, a drag strut or collector member should be provided and the new blocking extended as required beyond the end of the shear wall. The shear force is collected in the drag strut and transferred to the shear wall with more effective nailing or bolting. The new lumber must be dimensionally stable and cut to size.

Technique 3 (i.e., providing additional vertical-resisting elements) usually involves construction of additional interior shear walls or exterior buttresses. This alternative generally is more expensive than the other two because of the need for new foundations and for drag struts or other connections to collect the diaphragm shears for transfer to the new shear walls or buttresses.

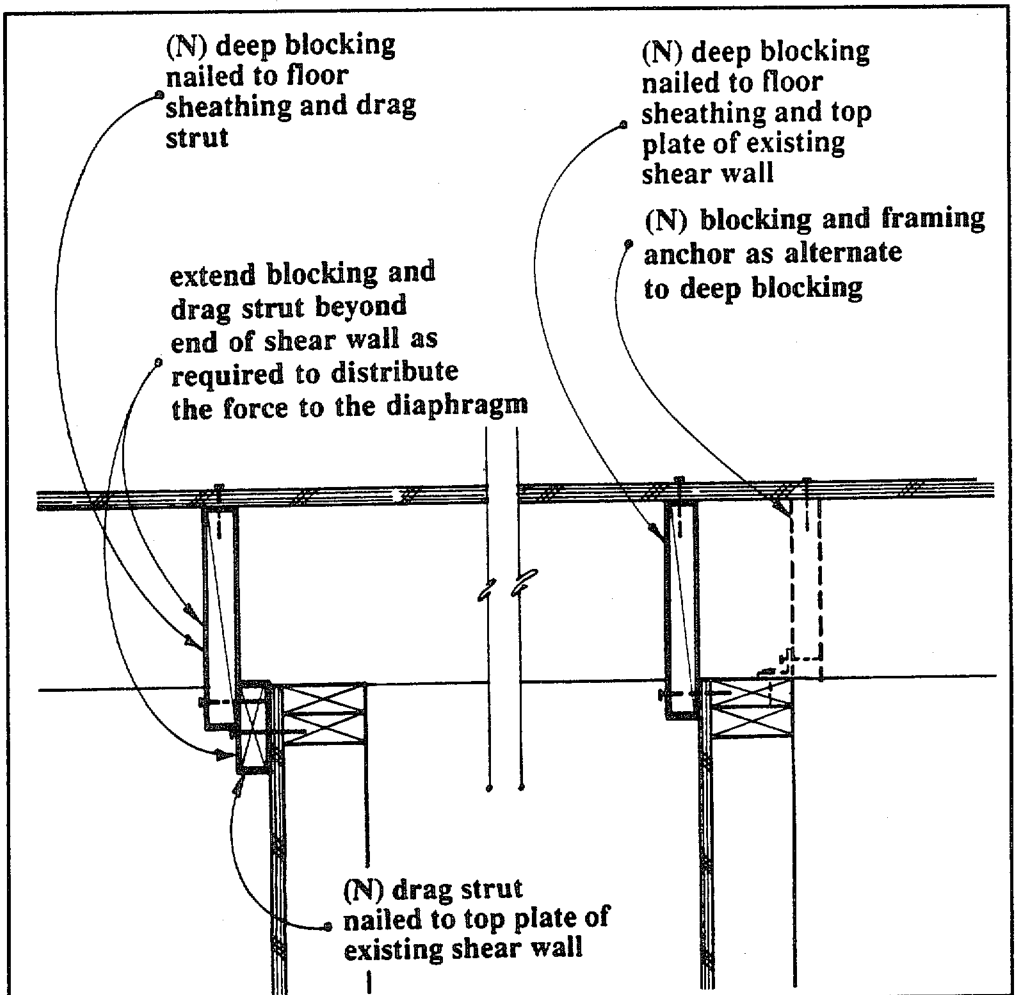

FIGURE 3.7.1.2b Strengthening the connection of a diaphragm to an interior shear wall (wall perpendicular to floor joist).

3.7.1.3 Strengthening Techniques for In-Plane Shear Transfer Capacity to Exterior Walls

Techniques. Deficient in-plane shear transfer capacity of a diaphragm to exterior shear walls or braced frames can be improved by:

1. Increasing the capacity of existing connections by providing additional nailing and/or bolting.
2. Reducing the local shear transfer stresses by distributing the forces from the diaphragm by providing chords or collector members to collect and distribute shear from the diaphragm to the shear wall or bracing (Figure 3.7.1.3).
3. Reducing the shear stress in the existing connection by providing supplemental vertical-resisting elements as discussed in Sec. 3.4.

Relative Merits. Inadequate in-plane shear transfer capacity at an exterior shear wall typically is a deficiency when large openings along the line of the wall exist. In this case, the shear force to be resisted per unit length of wall may be significantly greater than the shear force per unit length transferred from the diaphragm by the existing nailing or bolting. If the diaphragm and the shear walls have adequate shear capacity (as described for interior shear walls in Sec. 3.7.1.2), the solution requires transfer of the diaphragm shear to a collector member for distribution to the discontinuous shear walls. For timber shear walls parallel to the joists, the exterior joist usually is doubled up at the exterior wall and extended as a header over openings. This doubled joist can be spliced for continuity and used as a drag strut with shear transfer to the wall by means of metal clip anchors and nails or lag screws. Figure 3.7.1.3 shows an elevation of an existing wood stud shear wall with a large opening. If the resulting unit shears in the walls on either side of the opening are larger than the existing shear transfer capacity of the roof diaphragm (e.g., in this case, the capacity is governed by the existing nailing to the perimeter blocking or double joists), a collector member is required to collect the diaphragm shears and transfer them, at a higher shear stress, to the shear walls. In Figure 3.7.1.3, it is assumed that additional capacity is required for the existing shear walls and provided by new sheathing on the inside face. The assumed force path is from the roof sheathing to the blocking or double joists, from the blocking or joists to the exterior sheathing, from the exterior sheathing to the double plates at the top of the stud wall, and from the double plates to the collector members and the new sheathing. Adequate new or existing nailing must be provided at each of the above interfaces. The shear walls also must be checked for shear transfer at the foundation and the need for hold-down provisions to resist uplift from the additional forces. Note that, in the detail parallel to the joists, the existing double joists, if adequately spliced, can be utilized as a collector member. Similarly, if the existing double plates had been continuous over the opening, the collector member normal to the joists would not be required.

For steel frame buildings with discontinuous braced panels, the spandrel supporting the floor or roof framing may be used as a chord or collector member.

For discontinuous masonry, concrete or precast concrete shear walls parallel to the joists, the sheathing typically is nailed to a joist or ledger bolted to the wall. The joist or ledger can be spliced for continuity and supplementary bolting to the shear wall provided as required. For shear walls perpendicular to the joists, the sheathing may be nailed to discontinuous blocking between the ends of the joists. In this case, the chord or collector member may have to be provided on top of the diaphragm. This new member may be a continuous steel member bolted to the wall and nailed or lag screwed, with proper edge distance, to the diaphragm and also could be designed to provide out-of-plane anchorage as indicated in Figure 3.7.1.2b.

As discussed above with respect to interior wall connection deficiencies, providing additional vertical-resisting elements (Technique 3) is likely to be the most costly alternative unless it is being considered to correct other component deficiencies.

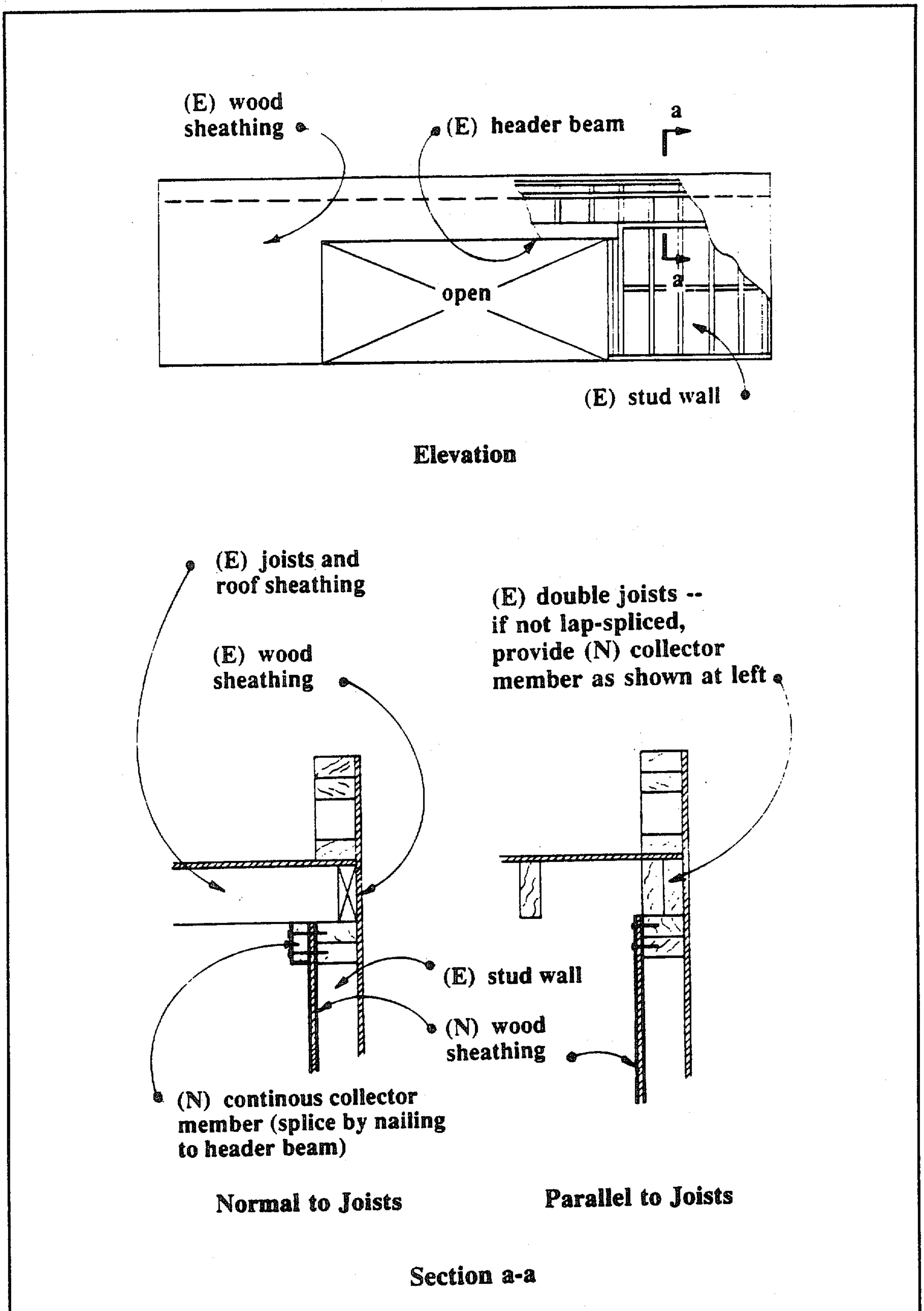

FIGURE 3.7.1.3 Strengthening an existing wood stud shear wall with a large opening.

3.7.1.4 Strengthening Techniques for Inadequate Out-of-Plane Anchorage

Techniques. Deficient out-of-plane anchorage capacity of wood diaphragms connected to concrete or masonry walls with wood ledgers can be improved by:

1. Increasing the capacity of the connection by providing steel straps connected to the wall (using drilled and grouted bolts or through bolts for masonry walls) and bolted or lagged to the diaphragm or roof or floor joists (Figures 3.7.1.4a, b, and c).

2. Increasing the capacity of the connections by providing a steel anchor to connect the roof or floor joists to the walls (Figure 3.7.1.4d).

3. Increasing the redundancy of the connection by providing continuity ties into the diaphragm (Figure 3.7.1.4a-d).

Relative Merits. An important condition to be addressed in retrofitting any existing heavy walled structure with a wood diaphragm is the anchorage of the walls for out-of-plane forces. Prior to the mid-1970s, it was common construction practice to bolt a 3x ledger to a concrete or masonry wall, install metal joist hangers to the ledger, drop in 2x joists, and sheath with plywood. The plywood that lapped the ledger would be nailed into the ledger providing both in-plane and out-of-plane shear transfer. The 1971 San Fernando earthquake caused many of these connections to fail. Out-of-plane forces stressed the ledgers in their weak cross-grain axis and caused many of them to split, allowing the walls to fall out and the roof to fall in. When retrofitting a masonry or concrete structure, this condition should be remedied by providing a positive connection between the concrete or masonry wall and wood diaphragm. Techniques 1 and 2 are, in general, equally cost-effective. In addition to correcting the ledger concerns, continuity ties need to be provided between diaphragm chords in order to distribute the anchorage forces well into the diaphragm. Joist hangers and glulam connections frequently have no tensile capacity, but this tensile capacity can be provided by installing tie rods bolted to adjacent joist or glulam framing (Figure 3.7.1.4e). These continuity ties provide a necessary redundancy in the connection of heavy walled structures to timber diaphragms.

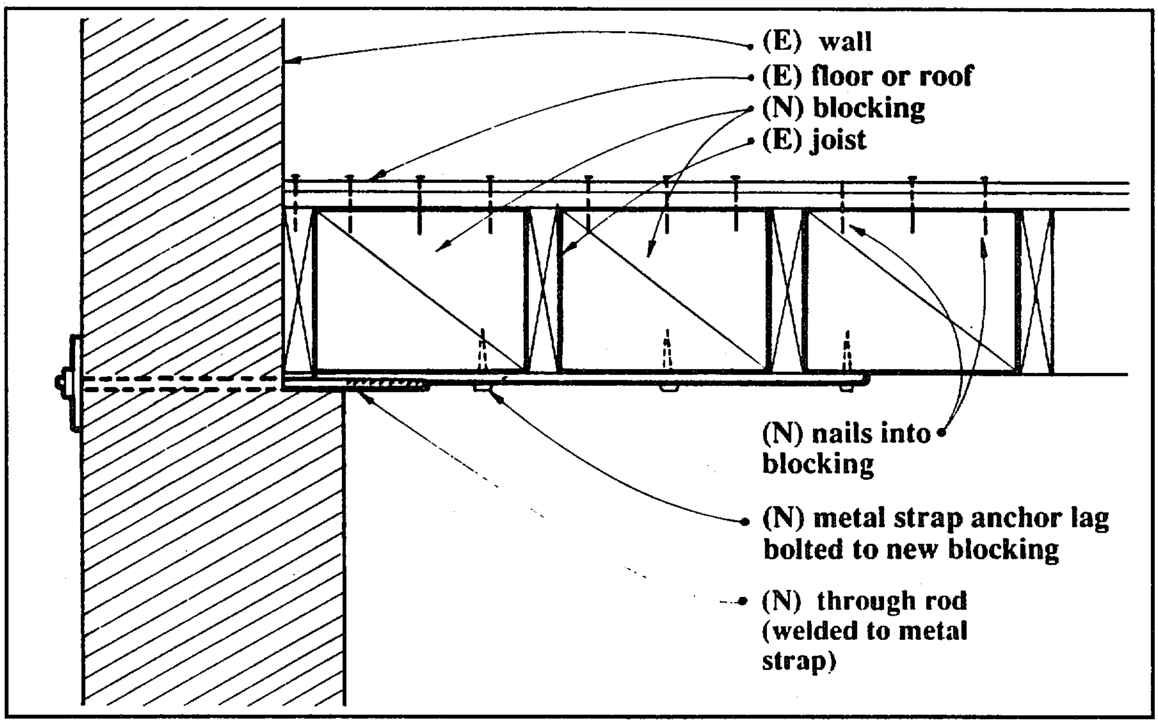

FIGURE 3.7.1.4a Strengthening out-of-plane connections of a wood diaphragm.

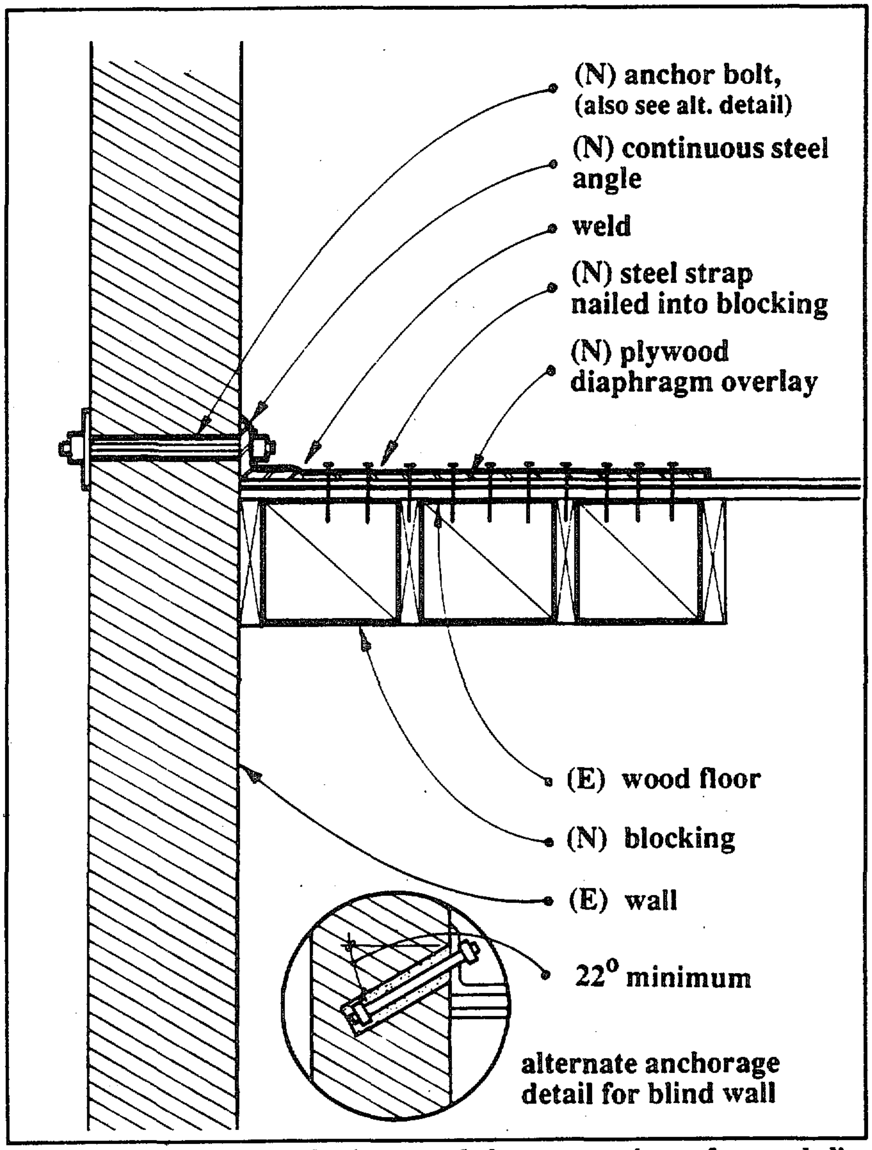

FIGURE 3.7.1.4b Strengthening out-of-plane connections of a wood diaphragm.

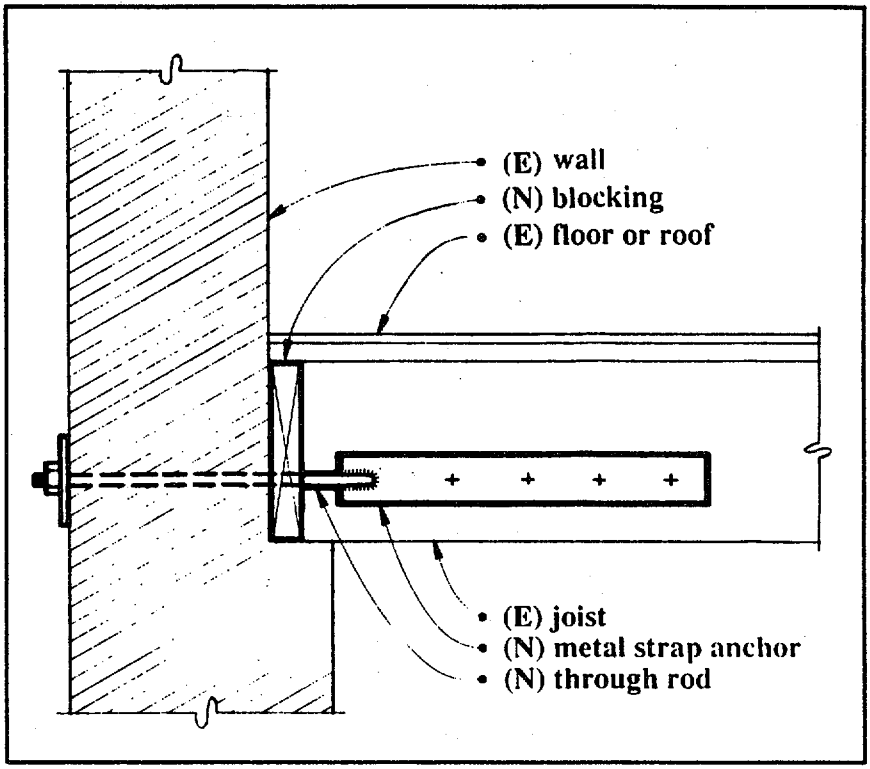

FIGURE 3.7.1.4c Strengthening out-of-plane connections of a wood diaphragm.

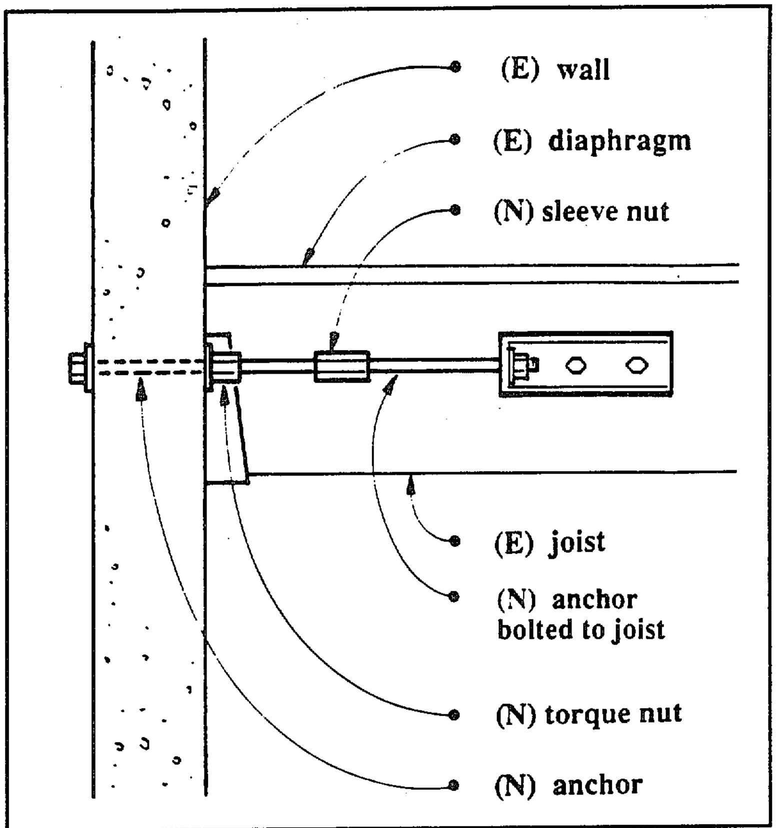

FIGURE 3.7.1.4d Strengthening out-of-plane connections of a wood diaphragm.

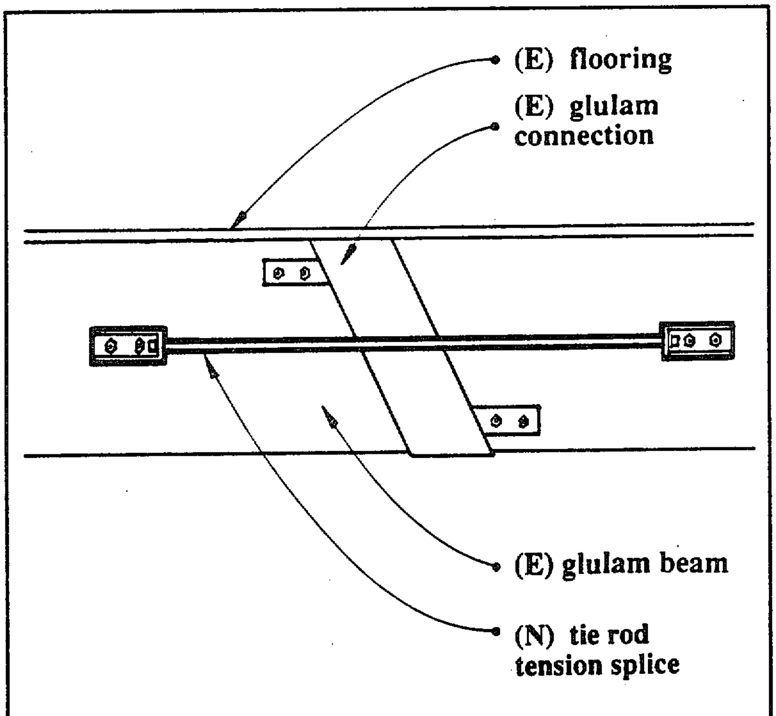

FIGURE 3.7.1.4e Strengthening tensile capacity of an existing glulam beam connection.

3.7.1.5 Strengthening Techniques for Interfloor Tensile Capacity

Techniques. Deficient tensile capacity of the connections of wood stud shear walls through diaphragms can be improved by:

1. Increasing the tensile capacity of the connections at the edge of the shear walls by providing metal connectors.

2. Reducing the overturning moments by providing supplemental vertical-resisting elements as discussed in Sec. 3.4.

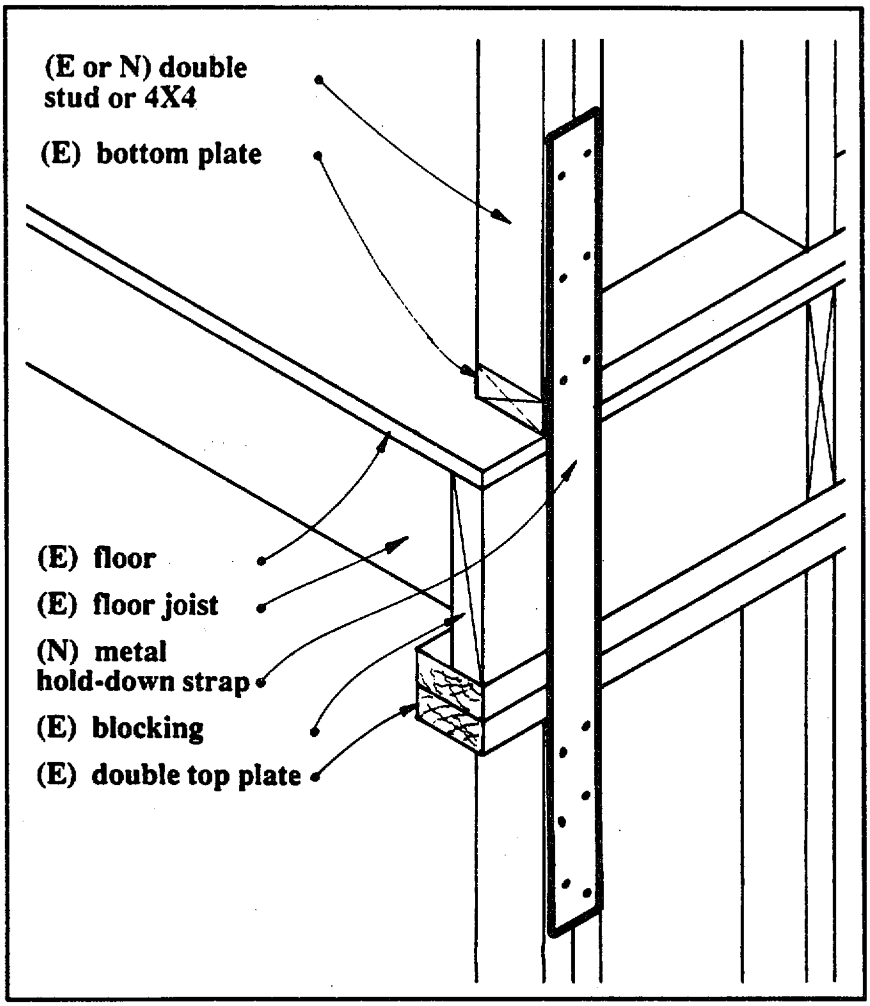

FIGURE 3.7.1.5a Strengthening the connection between shear walls using a metal strap.

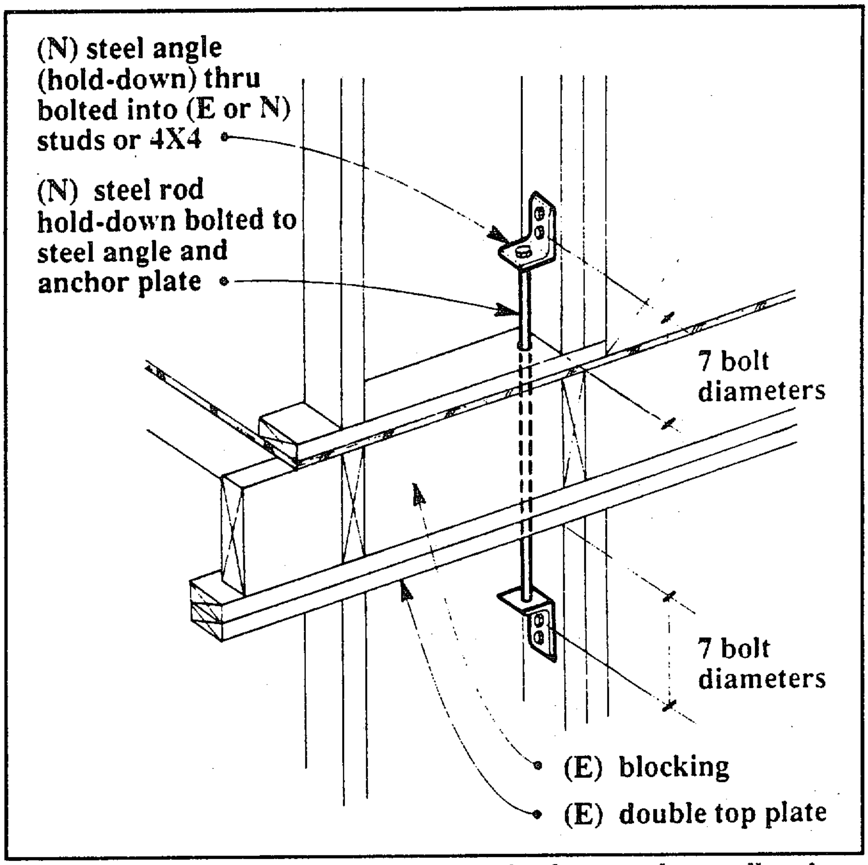

FIGURE 3.7.1.5b Strengthening the connection between shear walls using a hold-down.

Relative Merits. Typical wood stud framing has minimal capacity to transfer uplift forces from one shear wall to the shear wall below. At exterior walls, plywood sheathing generally is provided with a horizontal joint below the diaphragm to provide for "settling" shrinkage of the framing. Hence, minimal resistance to transfer uplift forces is provided unless continuity in the sheathing is provided by nailing top and bottom pieces to a common member (e.g., horizontal blocking or fascia as shown in Figure 3.5.1.3). The only resistance to uplift loads at exterior or interior shear walls may be the withdrawal capacity of the nails.

Metal straps or tie rods that tie the shear wall edge framing between floors (Figure 3.7.1.5c) are an economical approach to providing the prescribed tensile capacity. The wall finishes would be removed, a hole drilled or cut in the diaphragm or wall plates, and the connectors installed. Plywood shear walls should be adequately edge nailed to the double studs that are connected with the metal straps. For light timber structures, the metal straps may be of sheet metal and the sheathing can be nailed through the straps. When the straps are required to be of greater thickness, they may be recessed and drilled for nailing of the sheathing or, alternatively, the straps may be placed on the outside of the sheathing. See Figure 3.5.1.3 for typical splicing of sheathing and development of double top plates as chord or collector members. Figures 3.7.1.5a and b present two connection details where the shear wall on the upper floor does not align with the shear wall on a lower floor.

Technique 2 would be a viable alternative only if it is being considered to correct other component deficiencies (e.g., inadequate shear capacity in the existing walls).

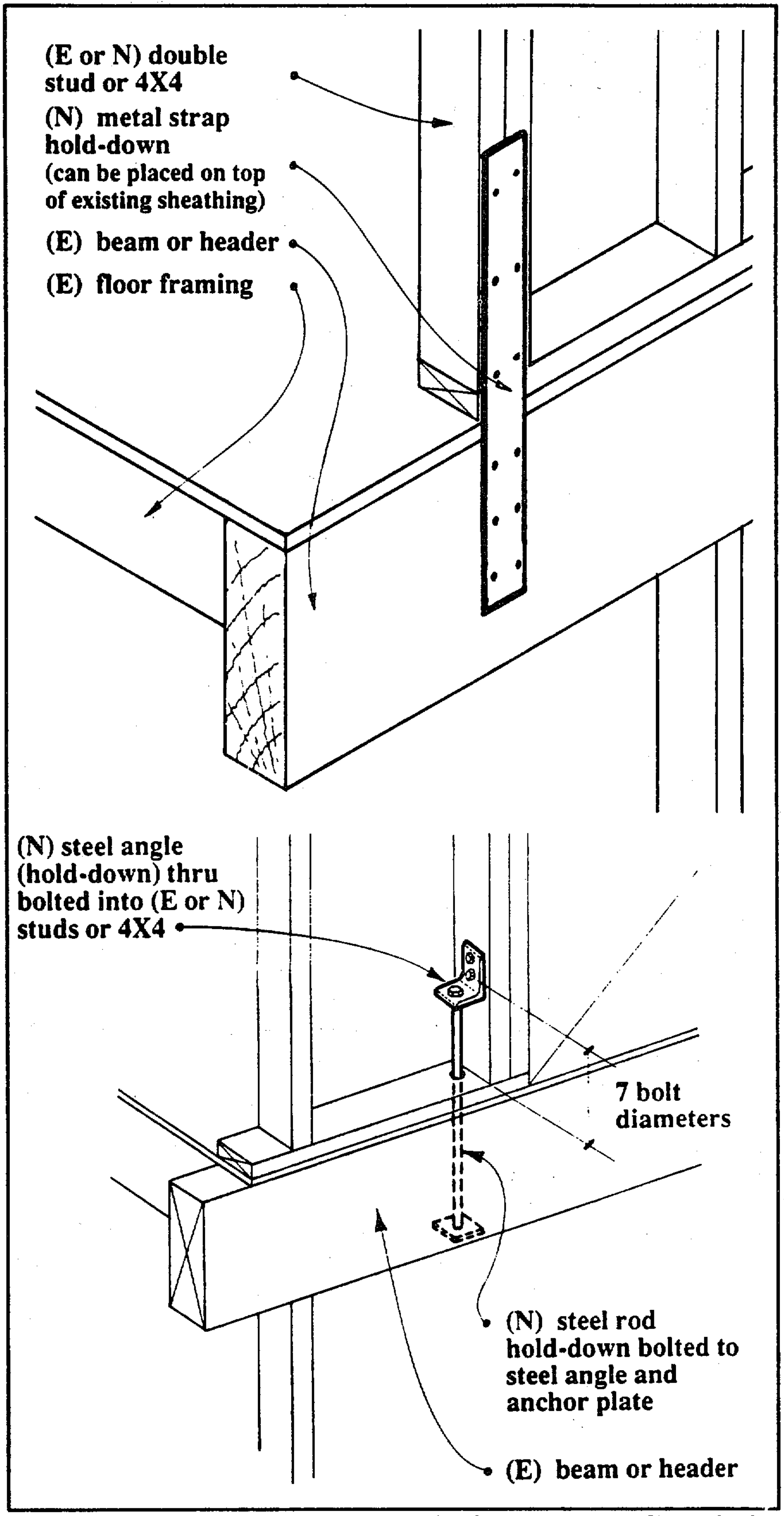

FIGURE 3.7.1.5c Strengthening shear wall uplift capacity at a discontinuity.

3.7.2 CONNECTIONS OF CONCRETE DIAPHRAGMS

3.7.2.1 Deficiencies

The principal deficiencies of the connections of concrete diaphragms to vertical-resisting elements such as shear walls or braced frames are:

- Inadequate in-plane shear transfer capacity and
- Inadequate anchorage capacity for out-of-plane forces in the connecting walls.

3.7.2.2 Strengthening Techniques for In-Plane Shear Wall Connections

Techniques. Deficient in-plane shear transfer capacity of a diaphragm to an interior shear wall or braced frame can be improved by:

1. Reducing the local stresses at the diaphragm-to-wall interface by providing collector members or drag struts under the diaphragm and connecting them to the diaphragm and the wall.

2. Increasing the capacity of the existing diaphragm-to-wall connection by providing additional dowels grouted into drilled holes.

3. Reducing the shear stresses in the existing connection by providing supplemental vertical-resisting elements as discussed in Sec. 3.4.

Relative Merits. Inadequate in-plane shear capacity of connections between concrete diaphragms and vertical-resisting elements usually occurs where large openings in the diaphragm exist adjacent to the shear wall (e.g., at stair wells) or where the shear force distributed to interior shear walls or braced frames exceeds the capacity of the connection to the diaphragm. If the walls and the diaphragm have sufficient capacity to resist the prescribed loads, the most cost-effective alternative to increase the connection capacity is likely to be providing additional dowels grouted into drilled holes (Technique 2). If the required connection capacity cannot be developed within the length of the shear wall, the addition of collector members (Technique 1) as indicated in Figure 3.7.2.2 is likely to be the most cost-effective alternative.

As previously discussed, reducing the forces in the deficient connection by providing supplemental vertical-resisting elements (Technique 3) is not likely to be the most cost-effective alternative (due to the probable need for new foundations and drag struts) unless it is being considered to correct other component deficiencies.

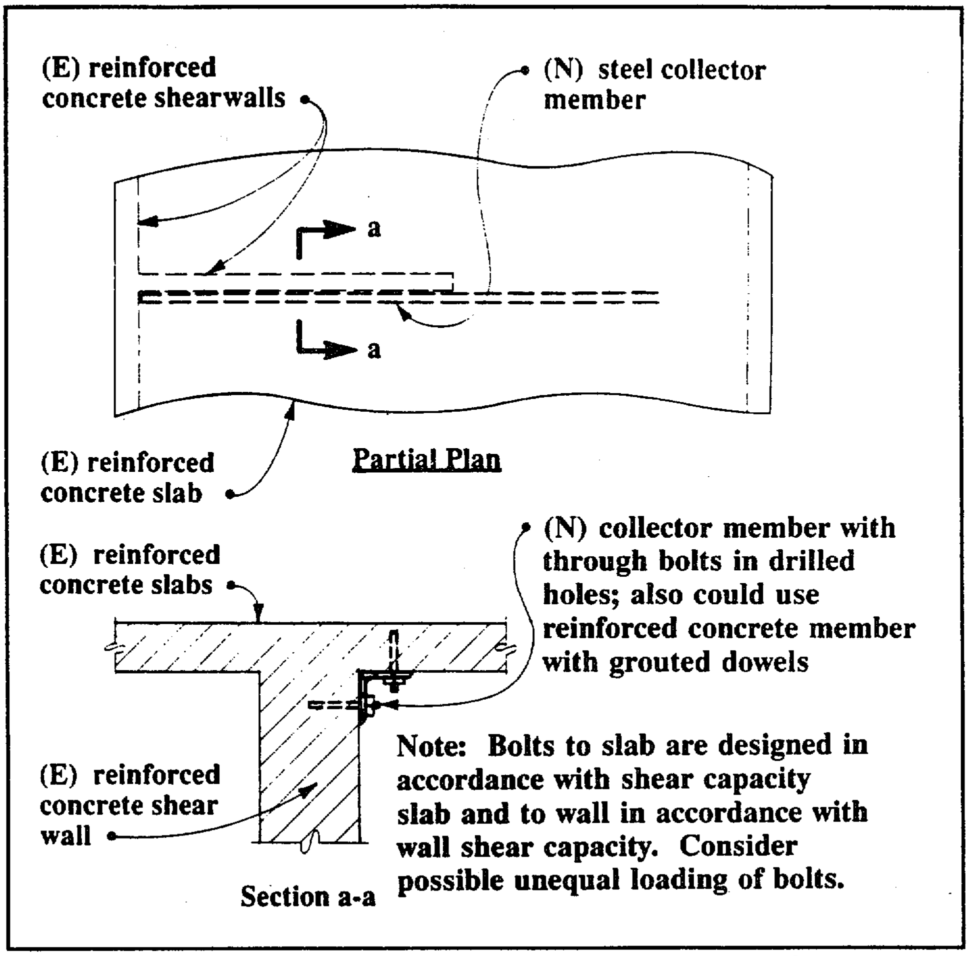

FIGURE 3.7.2.2 Use of a collector member to improve shear transfer from a concrete diaphragm.

3.7.2.3 Strengthening Techniques for Out-of-Plane Capacity

Techniques. Deficient out-of-plane anchorage capacity of connections of concrete diaphragms to concrete or masonry walls can be improved using one or both of the following techniques:

1. Increasing the capacity of the connection by providing additional dowels grouted into drilled holes.

2. Increasing the capacity of the connection by providing a new member above or below the slab connected to the slab with drilled and grouted bolts similar to that indicated in Figure 3.5.4.3 for providing a new diaphragm chord.

Relative Merits. The most cost-effective alternative generally is to provide additional dowels grouted into drilled holes (Technique 1). The holes are most efficiently drilled from the exterior through the wall and into the slab. Access to the exterior face of the wall is obviously required. When the exterior face is not accessible (e.g., when it abuts an adjacent building), providing a new member connected to the existing wall and slab (Technique 2) is likely to be preferred.

3.7.3 CONNECTIONS OF POURED GYPSUM DIAPHRAGMS

3.7.3.1 Deficiencies

The principal deficiencies of poured gypsum diaphragms are similar to those for concrete diaphragms:

- Inadequate in-plane shear transfer capacity and
- Inadequate anchorage capacity for out-of-plane forces in the connecting walls.

3.7.3.2 Strengthening Techniques

Techniques. If the gypsum diaphragm is in direct contact with the shear wall, it will be possible to improve the in-plane shear transfer by providing new dowels from the diaphragm into the shear wall similar to the details indicated in Figures 3.5.2.2 and 3.5.2.3. Alternative strengthening techniques for the deficiencies also include removal of the gypsum diaphragm and replacement with steel decking or the addition of a new horizontal bracing system designed to resist all of the seismic forces.

Relative Merits. As indicated in Sec. 3.5.3.2, allowable structural stresses for gypsum are very low and the additional strengthening that can be achieved is very limited. Further, the typical framing details (e.g., steel joist, bulb tee, and insulation board) are such that it is difficult to make direct and effective connections to the gypsum slab. For these reasons, the techniques involving removal and replacement or a new horizontal bracing system are likely to be the most cost-effective solutions except when the existing diaphragm is only marginally deficient.

3.7.4 CONNECTIONS OF PRECAST CONCRETE DIAPHRAGMS

3.7.4.1 Deficiencies

The principal deficiencies of the connections of precast concrete diaphragms to the vertical-resisting elements are:

- Inadequate in-plane shear transfer capacity and
- Inadequate anchorage capacity at the exterior walls for out-of-plane forces.

3.7.4.2 Strengthening Techniques for Precast Concrete Diaphragm Connections

Techniques. Deficient shear transfer or anchorage capacity of a connection of a precast concrete diaphragm to a concrete or masonry wall or a steel frame can be improved by:

1. Increasing the capacity of the connection by providing additional welded inserts or dowels placed in drilled or grouted holes.
2. Increasing the capacity of the connection by providing a reinforced concrete overlay that is bonded to the precast units and anchored to the wall with additional dowels placed in drilled and grouted holes (Figure 3.5.2.2).
3. Reducing the forces at the connection by providing supplemental vertical-resisting elements as discussed in Sec. 3.4.

Relative Merits. Precast concrete plank or tee floors that have inadequate connection capacity for transferring in-plane shear to vertical elements such as shear walls or braced frames can be strengthened by drilling intermittent holes in the precast units at the vertical element. When the floors are supported on steel framing, welded inserts (or studs) can be added and the holes grouted (Technique 1). When the floors are supported on concrete or masonry units, dowels can be inserted and grouted into the drilled holes. If the diaphragm contains prestressing strands, extreme care must be taken prior to drilling to avoid cutting the strands. A more costly alternative is to provide a reinforced concrete overlay that is bonded to the precast units and additional dowels grouted into holes drilled into the wall (Technique 2). This will require the stripping of the existing floor surface and raising the floor level by 2 to 3 inches, which will necessitate adjusting of nonstructural elements to the new floor elevation (e.g., stairs, doors, electrical outlets, etc.).

As previously discussed, reducing the shear forces in the deficient connection by providing supplemental vertical-resisting elements (Technique 3) is not likely to be the most cost-effective alternative (due to the probable need of new foundations and drag struts) unless it is being considered to correct other component deficiencies. This alternative also is not effective in reducing the out-of-plane forces unless the new vertical-resisting elements can be constructed so as to form effective buttresses for the existing walls.

3.7.5 CONNECTIONS OF STEEL DECK DIAPHRAGMS WITHOUT CONCRETE FILL

3.7.5.1 Deficiencies

For steel deck diaphragms without concrete fill, the principal deficiencies of their connections to the vertical-resisting elements such as shear walls, braced frames, or moment frames are:

- Inadequate in-plane shear capacity and
- Inadequate anchorage capacity for out-of-plane forces in walls.

3.7.5.2 Strengthening Techniques for Steel Deck Connections

Techniques. Deficient shear transfer or anchorage capacity of a connection of a steel deck diaphragm to a shear wall, braced frame, or moment frame can be improved by:

1. Increasing the capacity of the connection by providing additional welding at the vertical element.
2. Increasing the capacity of the connection by providing additional anchor bolts.
3. Increasing the capacity of the connection by providing concrete fill over the deck with dowels grouted into holes drilled into the wall.
4. Increasing the capacity of the connection by providing new steel members (Figure 3.7.5.2a) to effect a direct transfer of diaphragm shears to a shear wall.
5. Reducing the local stresses by providing additional vertical-resisting elements such as shear walls, braced frames, or moment frames as discussed in Sec. 3.4.

Relative Merits. Steel decking typically is supported by metal framing, by steel angle, or by channel ledgers bolted to concrete or masonry walls. If the deficiency is in the connection and not the diaphragm, the most cost-effective alternative is to increase the welding of the decking to the steel member or ledger to at least the capacity of the diaphragm. If supported by a ledger, the capacity of the ledger connections to the concrete or masonry wall also may have to be improved; this is most effectively done by providing additional bolts in drilled and grouted holes.

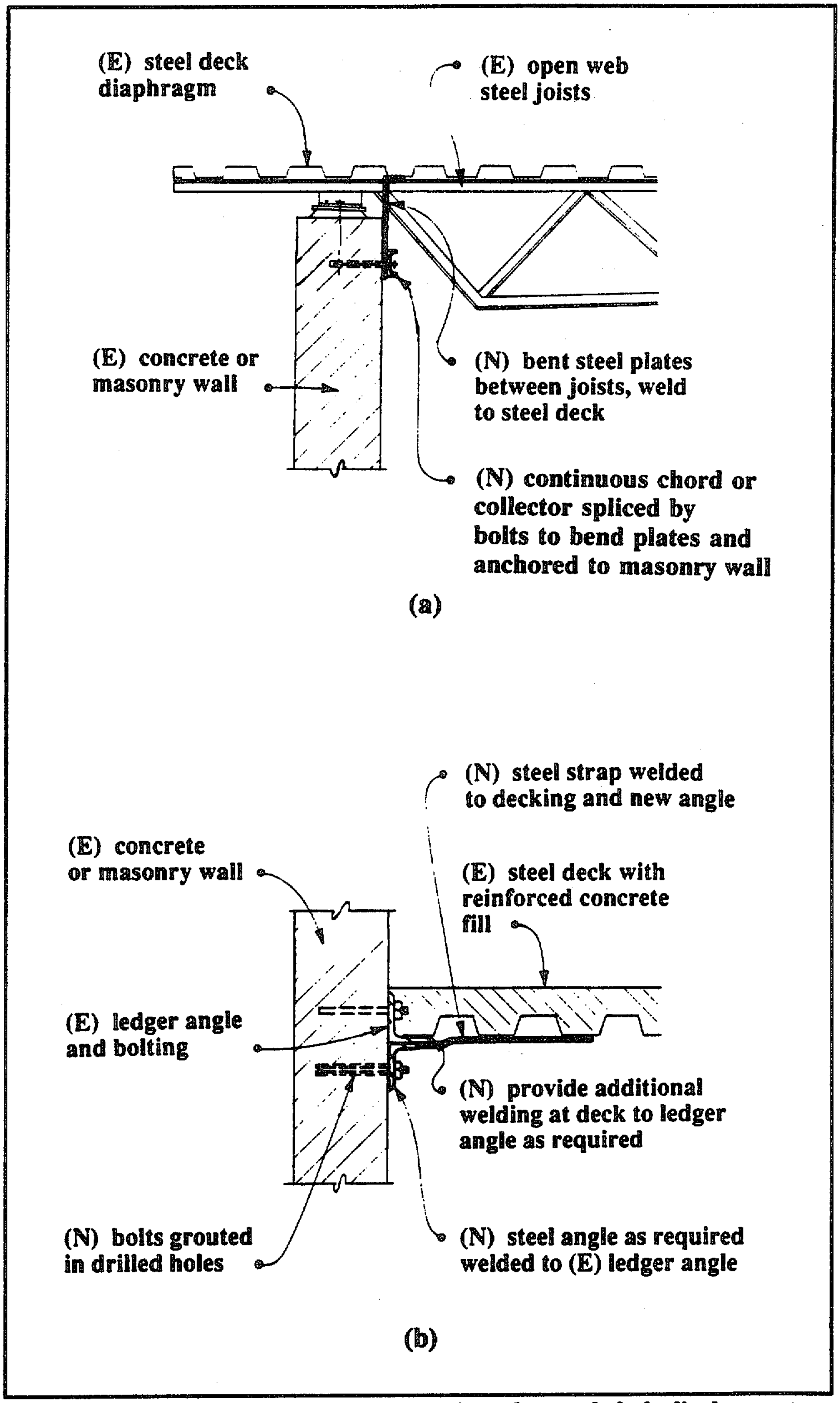

FIGURE 3.7.5.2 Strengthening the connection of a steel deck diaphragm to a concrete or masonry wall.

If the decking is being reinforced by filling with reinforced concrete, the most effective alternative will be to drill and grout dowels into the adjacent concrete or masonry wall and lap with reinforcing steel in the new slab. In some cases it may be feasible to use the existing steel support member at the wall as a collector as shown in Figure 3.7.5.2b. In this figure the capacity of the existing decking has been increased by additional welding to the ledger angle and the addition of a reinforced concrete fill. Reinforcement dowels are welded to the angle that functions as a collector member and the shear forces are transferred to the wall by the existing and new anchor bolts, as required.

Steel deck roof diaphragms may be supported on open web steel joists that rest on steel bearing plates at the top of concrete or masonry walls. In existing buildings that have not been properly designed for resisting lateral loads, there may not be a direct path for the transfer of diaphragm shears to the vertical walls, particularly when the decking span is parallel to the wall. As shown in Figure 3.7.5.2a, new steel elements (i.e., bent plates) can be provided between the joists for direct connection to the decking. A continuous member also can be provided to function as a chord or collector member. As noted above, strengthening a steel deck diaphragm connection to the vertical-resisting elements is effective only if the body of the diaphragm has adequate capacity to resist the design lateral forces. If the diaphragm does not have adequate capacity it needs to be strengthened as discussed in Sec. 3.5.5.

As previously discussed, reducing the shear transfer forces in the deficient connection by providing supplemental vertical-resisting elements (Technique 4) is not likely to be the most cost-effective alternative (due to the probable need of new foundations and drag struts) unless it is being considered to correct other component deficiencies. Further, in order to reduce out-of-plane wall forces, the new vertical elements would be required to act as buttresses to the existing walls.

3.7.6 CONNECTIONS OF STEEL DECK DIAPHRAGMS WITH CONCRETE FILL

3.7.6.1 Deficiency

The principal deficiency of a connection of a steel deck diaphragm with concrete fill to the vertical-resisting elements such as shear walls, braced frames, or moment frames is the in-plane shear capacity or anchorage capacity for out-of-plane forces in walls.

3.7.6.2 Strengthening Techniques for Steel Deck Connections

Techniques. Deficient shear capacity or anchorage of a connection of a steel deck diaphragm to a shear wall, braced frame, or moment frame can be improved by:

1. Increasing the shear capacity by drilling holes through the concrete fill, and providing additional shear studs welded to the vertical elements through the decking (Figure 3.5.5.2a).

2. Increasing the capacity of the connection by providing additional anchor bolts (drilled and grouted) connecting the steel support to the wall.

3. Increasing the capacity of the connection by placing dowels between the existing wall and diaphragm slab.

4. Reducing the local stresses by providing additional vertical-resisting elements such as shear walls, braced frames, or moment frames as discussed in Sec. 3.4.

Relative Merits. If the deficiency is in both the connection of the diaphragm to the ledger and the ledger to the shear wall, the most cost-effective alternative may be to provide a direct force transfer from the slab to the wall by installing dowels (Technique 3). This is accomplished by removing the concrete to expose the diaphragm slab reinforcement, drilling holes in the wall, laying in dowels, and grouting and reconstructing the diaphragm slab. If the deficiency is in the slab-to-supporting steel member connection, Technique 1 is preferred. If the deficiency is in the steel ledger to the wall connection, Technique 2 is preferred. Figure 3.7.5.2b illustrates a technique for strengthening a steel deck diaphragm connection to a concrete or masonry wall. In this figure, it is assumed that the existing decking with concrete fill has adequate capacity for the design loads, but the connection to the wall is deficient for in-plane shear and out-of-plane anchorage forces. In the figure, the in-plane shear is assumed to be transferred from the decking to the existing ledger angle with additional welding (if required). The new angles, bolted to the wall and welded to the ledger angle, provide the necessary additional shear transfer capacity. The new steel straps, welded to the new angles and to the underside of the decking, provide the additional out-of-plane anchorage capacity. When the new dowels or anchor bolts are to be attached to existing thin concrete walls (e.g., precast tees or other thin ribbed concrete sections), through bolts or threaded rods are required to provide adequate anchorage or doweling to the diaphragm. If the vertical-resisting elements are steel braced frames or steel moment frames, the increase in connection capacity obviously would be achieved through additional welding and supplemental reinforcing members as required.

As previously discussed, reducing the forces in the deficient connection by providing supplemental vertical-resisting elements (Technique 4) is unlikely to be the most cost-effective alternative (due to the probable need of new foundations and drag struts) unless it is being considered to correct other component deficiencies. Further, in order to reduce out-of-plane wall forces, the new vertical elements would be required to act as buttresses to the existing walls.

3.7.7 CONNECTIONS OF HORIZONTAL STEEL BRACING

3.7.7.1 Deficiencies

The two primary deficiencies in the connection capacity of horizontal steel braces to vertical-resisting elements such as shear walls or braced frames are:

- Inadequate in-plane shear transfer capacity and
- Inadequate anchorage capacity when supporting concrete or masonry walls for out-of-plane forces.

3.7.7.2 Strengthening Techniques for In-Plane Shear Transfer Capacity

Techniques. Deficient shear transfer capacity of connections of horizontal steel bracing systems to shear walls or braced frames can be improved by:

1. Increasing the capacity by providing larger or more bolts or by welding.
2. Reducing the stresses by providing supplemental vertical-resisting elements such as shear walls or braced frames as discussed in Sec. 3.4.

Relative Merits. The first alternative of providing larger or more bolts between the horizontal brace members and the concrete or masonry shear wall or providing additional welding when connecting to a steel braced frame generally will be the most cost-effective. Collectors along the wall may be required to distribute the concentrated brace shear along the wall to allow for adequate bolt spacing.

As previously discussed, reducing the forces in the deficient connection by providing supplemental vertical-resisting elements (Technique 2) is not likely to be the most cost-effective alternative unless it is being considered to correct other component deficiencies.

3.7.7.3 Strengthening Technique for Out-Of-Plane Anchorage

Technique. Deficient out-of-plane anchorage capacity of connections between horizontal steel bracing systems and concrete or masonry shear walls can be improved by increasing the capacity of the connection by providing additional anchor bolts grouted in drilled holes and by providing more bolts or welding to the bracing members.

3.8 VERTICAL ELEMENT TO FOUNDATION CONNECTIONS

Seismic inertial forces originate in all elements of buildings and are delivered through structural connections to horizontal diaphragms. The diaphragms distribute these forces to vertical elements that transfer the forces to the foundation and the foundation transfers the forces into the ground.

An adequate connection between the vertical elements and the foundation is essential to the satisfactory performance of a strengthened structure. The connections must be capable of transferring the in-plane lateral inertia forces from the vertical elements to the foundations and of providing adequate capacity for resisting uplift forces caused by overturning moments.

3.8.1 CONNECTIONS OF WOOD STUD SHEAR WALLS

3.8.1.1 Deficiencies

The principal deficiencies in the connection of wood stud shear walls to their foundations are:

- Inadequate shear capacity of the anchorage,
- Inadequate shear capacity of cripple stud walls, and
- Inadequate uplift capacity.

3.8.1.2 Strengthening Techniques for Inadequate Anchorage Shear Capacity

Techniques. Deficient shear capacity of the connection of a wood stud wall to its foundation can be improved using one or both of the following alternatives:

1. Increasing the shear capacity by providing new or additional anchor bolts between the sill plate and the foundation (Figure 3.8.1.2a).

2. Increasing the shear capacity by providing steel angles or plates with anchor bolts connecting them to the foundation and bolts or lag screws connecting them to the sill plate or wall (Figure 3.8.1.2b).

Relative Merits. Lack of adequate anchorage of the walls to the foundation can cause poor seismic performance of wood frame structures. Although most older wood frame structures were not designed for seismic loads, they have performed extremely well in past earthquakes provided they were bolted to their foundation. This good performance may be attributed to their light weight, ductile connections, and redundant load paths provided they are bolted to the foundations.

If the walls are not bolted to the foundation it is relatively simple to provide adequate anchorage. Providing bolts through the sill plates in the foundation (Technique 1) is typically the best approach. If a crawl space exists, the bolts can be installed easily at regular intervals. If the walls sit directly on the foundation without floor joists (e.g., a slab on grade), access through the wall covering (e.g., gypsum board) is required and the wall surface subsequently must be patched. If the crawl space is not deep enough for vertical holes to be drilled through the sill plate, the addition of connection plates or angles (Technique 2) may be a more viable alternative.

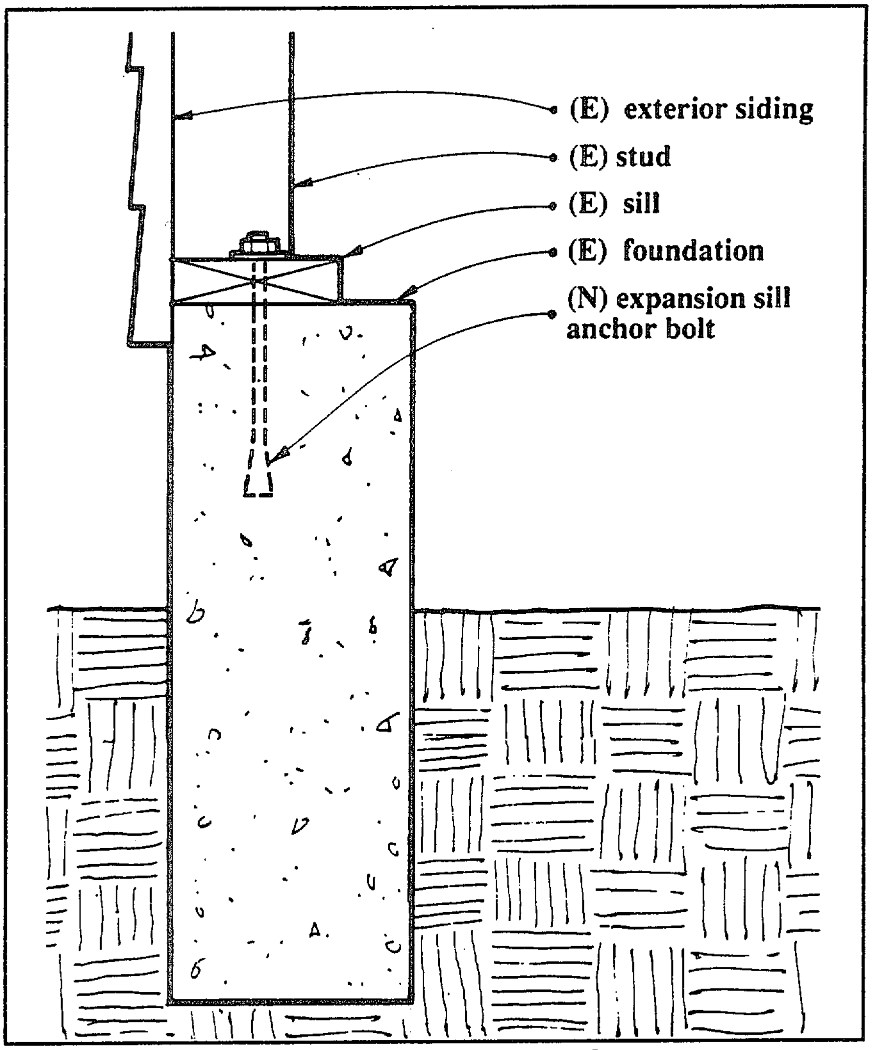

FIGURE 3.8.1.2a Providing wall to foundation anchors.

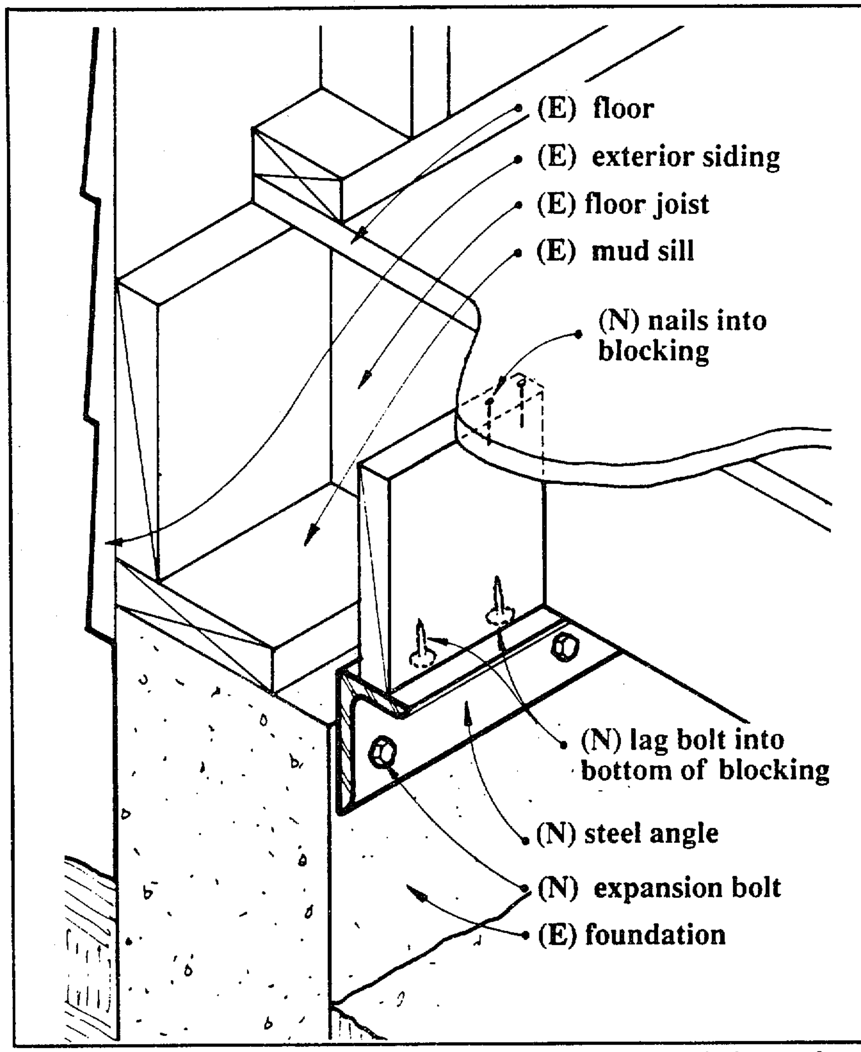

FIGURE 3.8.1.2b Alternate detail for providing wall to foundation anchors.

3.8.1.3 Strengthening Techniques for Cripple Stud Walls

Techniques. Weak cripple stud walls also are a significant reason for damage to wood frame structures. Cripple stud perimeter walls are a frequent construction technique used to support the first floor of a wood structure a short distance above the ground on sloping sites or to provide a crawl space under the floor framing. The exterior face may be finished with wood or metal siding or plaster while the studs on the inside usually remain exposed.

Strengthening of the cripple stud walls is relatively simple. Plywood sheathing is nailed to the cripple studs (usually on the inside). The top edge of the plywood is nailed into the floor framing and the bottom edge is nailed into the sill plate (Figure 3.8.1.3). The sill plate also must be anchored to the foundation.

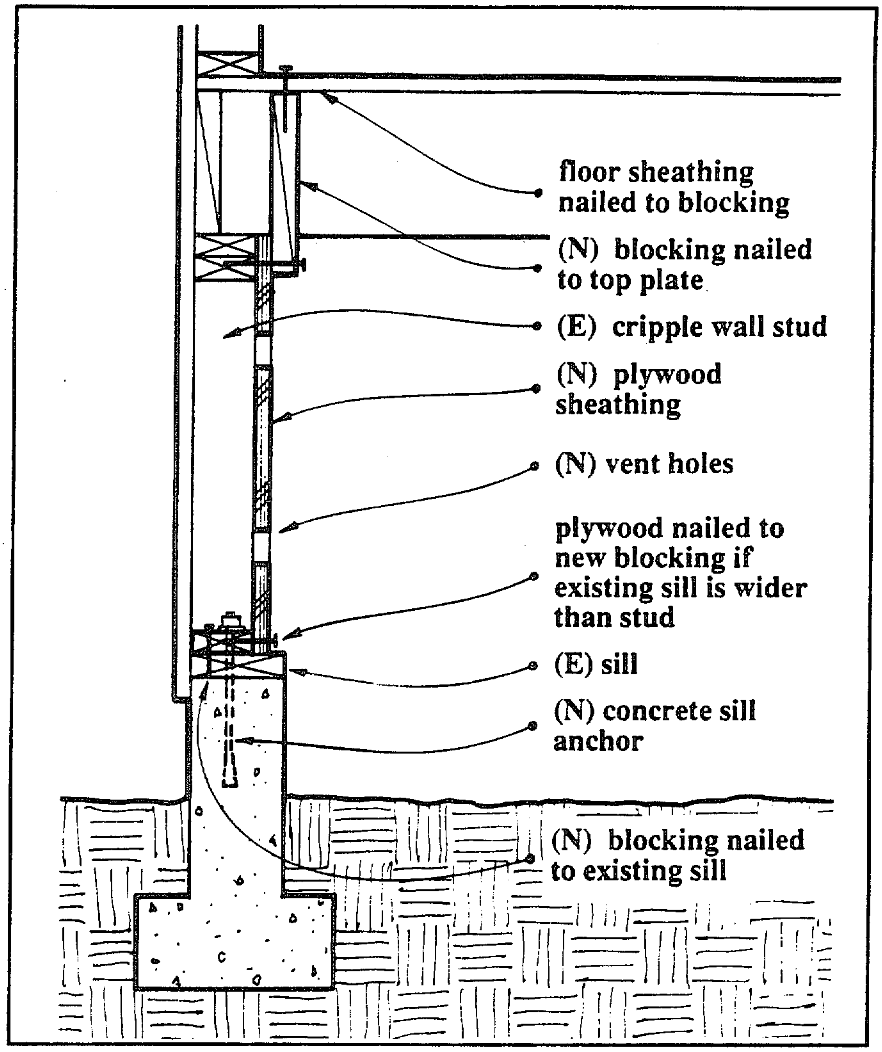

FIGURE 3.8.1.3 Strengthening a cripple stud wall.

3.8.1.4 Strengthening Techniques for Uplift Capacity

Deficient uplift capacity of the connections of wood shear walls to their foundations can be improved by:

1. Increasing the capacity by providing steel hold-downs bolted to the wall and anchored to the concrete (Figure 3.8.1.4).

2. Reducing the uplift requirement by providing supplemental shear walls (as discussed in Sec. 3.4.).

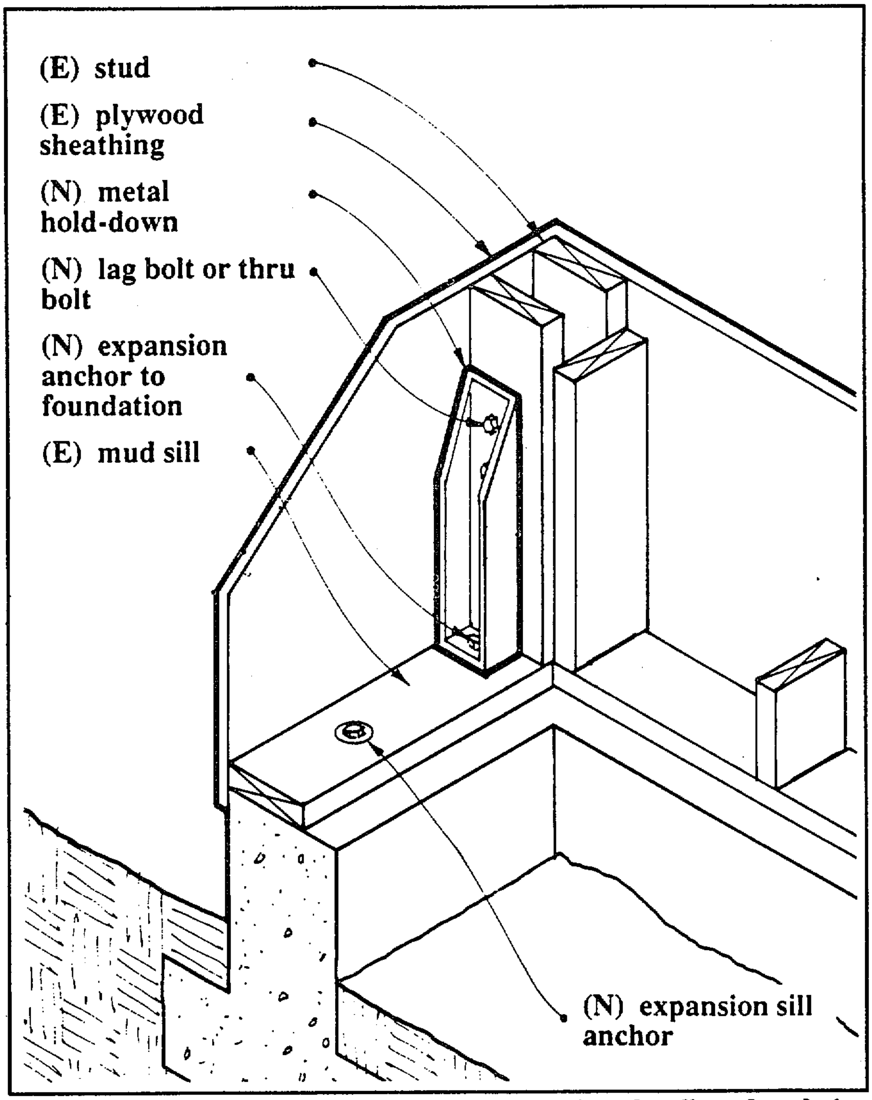

FIGURE 3.8.1.4 Strengthening the uplift capacity of wall to foundation connection.

3.8.2 CONNECTIONS OF METAL STUD SHEAR WALLS

The connections of metal stud walls to the foundations can be strengthened in the same way as discussed above for wood stud walls (e.g., by adding welding, bolting, and screws where appropriate).

3.8.3 CONNECTIONS OF PRECAST CONCRETE SHEAR WALLS

3.8.3.1 Deficiencies

The principal deficiencies of the connections of precast concrete shear walls to the foundation are:

- Inadequate capacity to resist in-plane or out-of-plane shear forces and
- Inadequate hold-down capacity to resist seismic overturning forces.

3.8.3.2 Strengthening Techniques for Shear Capacity

Techniques. Deficient shear capacity of the connections of precast concrete shear walls to the foundation can be improved by:

1. Increasing the capacity of the connection by providing a new steel member connecting the wall to the foundation or the ground floor slab (Figure 3.8.3.2).

2. Increasing the capacity of the connection by adding a new thickness of concrete (either cast-in-place or shotcrete) placed against the precast wall doweling into the existing foundation or ground floor slab.

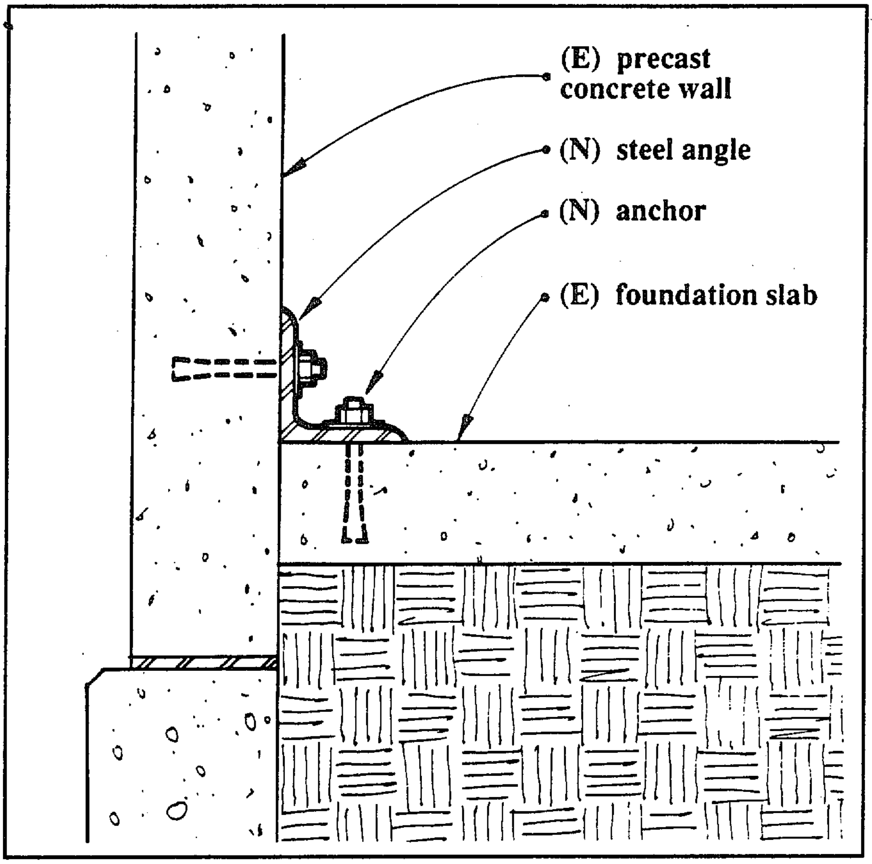

FIGURE 3.8.3.2 Strengthening of a precast concrete wall to foundation connection.

Relative Merits. Early precast concrete wall construction frequently had minimal lateral connection capacity at the foundation. These connections usually can be strengthened most economically by attaching a steel member to the wall and the floor slab or foundation with drilled and grouted anchors or expansion bolts (Technique 1). Care must be taken to place bolts and/or dowels a sufficient distance away from concrete edges to prevent spalling under load. A more costly alternative involves thickening the precast wall with a minimum of 4 inches of new reinforced concrete, either cast-in-place or shotcrete. The new concrete must be anchored to the precast wall and must extend above the base of the wall high enough to develop new dowels drilled into the foundation. The existing foundation then must be checked for the additional load.

3.8.3.3 Strengthening Techniques for Hold-down Capacity

Techniques. Deficient hold-down capacity of the connections of precast concrete shear walls to the foundation can be improved by:

1. Increase the hold-down capacity by removing concrete at the edge of the precast unit to expose the reinforcement, provide new drilled and grouted dowels into the foundation, and pour a new concrete pilaster.

2. Reduce the uplift forces by providing supplemental vertical-resisting elements such as shear walls or braced frames as discussed in Sec. 3.4.